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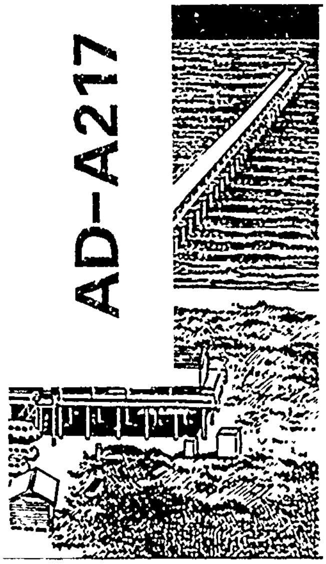

TECHNICAL REPORT CERC-89-18

NOYO RIVER AND HARBOR, CALIFORNIA DESIGN FOR WAVE PROTECTION SUPPLEMENTAL TESTS

Coastal Model Investigation

by

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Waterways Experiment Station, Corps of Engineers
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December 1989

Final Report

Approved For Public Release, Distribution Unlimited

Prepared for US Army Engineer District, San Francisco
San Francisco, California 94105-1905

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1a REPORT SECURITY CLASSIFICATION Unclassified	1b RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY	3 DISTRIBUTION/AVAILABILITY OF REPORT Approved for public release, distribution unlimited.
2b DECLASSIFICATION/DOWNGRADING SCHEDULE	
4 PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Report CERC-89-18	5 MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION USAEWES, Coastal Engineering Research Center 6b OFFICE SYMBOL *(If applicable)*	7a NAME OF MONITORING ORGANIZATION
6c. ADDRESS *(City State, and ZIP Code)* 3909 Halls Ferry Road Vicksburg, MS 39180-6199	7b ADDRESS *(City, State, and ZIP Code)*
8a NAME OF FUNDING/SPONSORING ORGANIZATION US Army Engineer District, San Francisco 8b OFFICE SYMBOL *(If applicable)*	9 PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS *(City, State, and ZIP Code)* San Francisco, California 94105-1905	10 SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO / PROJECT NO / TASK NO. / WORK UNIT ACCESSION NO

11 TITLE *(Include Security Classification)*
Noyo River and Harbor, California, Design for Wave Protection, Supplemental Tests, Coastal Model Investigation

12. PERSONAL AUTHOR(S)
Bottin, Robert R., Jr.; Mize, Marvin G.

13a. TYPE OF REPORT	13b TIME COVERED	14 DATE OF REPORT *(Year, Month, Day)*	15 PAGE COUNT
Final report	FROM Nov 88 TO Feb 89	December 1989	60

16 SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17 COSATI CODES			18 SUBJECT TERMS *(Continue on reverse if necessary and identify by block number)*
FIELD	GROUP	SUB-GROUP	Breakwaters Harbors, California Hydraulic models. (AU) Noyo River and Harbor, California Wave protection

19. ABSTRACT *(Continue on reverse if necessary and identify by block number)*

An existing 1:75-scale undistorted hydraulic model of Noyo River and Harbor was reactivated to determine wave conditions in the entrance (for 14-ft incident waves) as a result of various breakwater configurations in Noyo Cove. The model reproduced the river from its mouth to a point approximately 15,000 ft upstream, both Noyo Harbor and Dolphin Isle Marina located on the south bank, approximately 3,500 ft of the California shoreline on each side of the river mouth, Noyo Cove, and sufficient offshore area in the Pacific Ocean to permit generation of the required test waves. A 45-ft-long wave generator and an automated data acquisition and control system were utilized in model operation. It was concluded from the model investigation that:

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20 DISTRIBUTION/AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT ☐ DTIC USERS	21 ABSTRACT SECURITY CLASSIFICATION Unclassified
22a. NAME OF RESPONSIBLE INDIVIDUAL	22b TELEPHONE *(Include Area Code)* / 22c OFFICE SYMBOL

DD Form 1473, JUN 86 *Previous editions are obsolete.* SECURITY CLASSIFICATION OF THIS PAGE

Unclassified

19. ABSTRACT (Continued).

a. Existing conditions are characterized by very rough and turbulent wave conditions in the Noyo River jettied entrance for 15-sec, 14-ft incident design wave conditions. Waves with heights ranging from 9.3 to 11.7 ft will occur in the entrance, depending on direction of wave approach.

b. Of the test plans involving a shore-connected outer north breakwater and a detached inner breakwater (Plans 1 through 14), Plan 14 (300-ft-long outer and 250-ft-long inner breakwaters) will meet the established 6.0-ft wave-height criterion in the existing entrance for design wave conditions from all directions. Wave heights in the entrance for waves from the predominant west-northwest direction will be 3.5 ft or less.

c. Incremental removal of the Plan 14 outer breakwater (Plans 27 through 31) indicated that the 250-ft-long inner breakwater alone (Plan 31) would meet the established criterion for design wave conditions from all directions. Wave heights up to 5.8 ft will exist in the entrance for waves from the predominant west-northwest direction.

d. Neither the outer shore-connected north breakwater and outer detached south breakwater (Plan 15) nor the outer detached south breakwater plans (Plan 16 through 18) will meet the established wave-height criterion for design wave conditions. Wave heights will range from 7.4 to 11.2 ft in the existing entrance for these plans.

e. Of the improvement plans involving a curved breakwater seaward of the existing entrance (Plans 19 through 22), the 450-ft-long structure of Plan 22 will meet the established wave-height criterion for design wave conditions from all directions. Wave heights of 6.0 ft will exist in the entrance for waves from the predominant west-northwest direction.

f. Of the improvement plans involving two inner detached breakwaters (Plans 23 through 26), the 375-ft-long north structure and 250-ft-long south breakwater (Plan 26) will meet the established wave-height criterion in the entrance for design wave conditions from all directions. Wave heights in the existing entrance will be 5.8 ft for waves from the predominant west-northwest direction.

PREFACE

A request for additional testing on the existing Noyo River and Harbor model was initiated by the US Army Engineer District, San Francisco (SPN). Authorization for the US Army Engineer Waterways Experiment Station's (WES) Coastal Engineering Research Center (CERC) to perform the study was subsequently granted and funds were authorized by SPN on 12 September 1988 and 16 May 1989. Engineering support was provided SPN by the US Army Engineer District, Los Angeles (SPL).

Model testing was conducted at WES during the period from November 1988 through February 1989 by personnel of CERC under the direction of Dr. J. R. Houston and Mr. C. C. Calhoun, Jr., Chief and Assistant Chief, CERC, respectively; and under direct supervision of Messrs. C. E. Chatham, Jr., Chief, Wave Dynamics Division, and D. G. Outlaw, Chief, Wave Processes Branch. The tests were conducted by Mr. M. G. Mize, Civil Engineering Technician, under the supervision of Mr. R. R. Bottin, Jr., Project Manager. This report was prepared by Messrs. Bottin and Mize.

During the course of the investigation, liaison was maintained by means of conferences, telephone communications, and monthly progress reports. Messrs. Arijs Rakstins, Kerry Guy, and Richard Lou, SPN, and Art Shak, SPL, visited WES to observe model operation and participate in a conference during the course of the study; and Mr. Bottin visited the SPN office and the city of Fort Bragg, California, to present model test results after completion of the study.

COL Dwayne G. Lee, EN, was Commander and Director of WES during the conduct of this investigation and COL Larry B. Fulton, EN, was Commander and Director of WES during the preparation and publication of this report. Dr. Robert W. Whalin was Technical Director.

Initial test results for the model were reported in WES Technical Report CERC-88-15, "Noyo River and Harbor, California, Design for Wave and Surge Protection; Coastal Model Investigation," dated September 1988. Test results for supplemental wave conditions are reported herein.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	metres
miles (US statute)	1.609344	kilometres
pounds (force)	4.4482224	newtons
square feet	0.09290304	square metres
square miles (US statute)	2.589988	square kilometres
yards	0.9144	metres

NOYO RIVER AND HARBOR, CALIFORNIA

DESIGN FOR WAVE PROTECTION

SUPPLEMENTAL TESTS

Coastal Model Investigation

PART I: INTRODUCTION

The Prototype

1. Noyo River and Harbor are located on the California coast in Mendocino County, approximately 135 miles* north of San Francisco and 87 miles south of Eureka (Figure 1). The shoreline in the locality consists of broken, irregular cliffs about 40 to 80 ft high with numerous rocks extending several hundred yards offshore. Small pocket beaches are found at the heads of coves

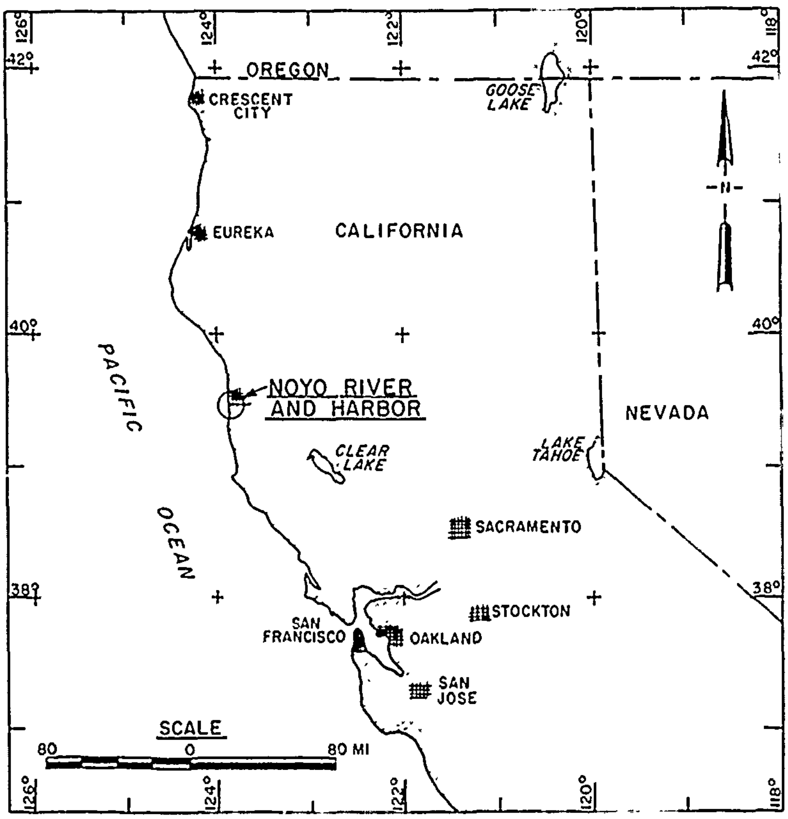

Figure 1. Project location

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

in the immediate vicinity. The Noyo River empties into Noyo Cove which is approximately 1,800 ft wide, north to south, and 2,000 ft long, east to west.

2. The existing Noyo River and Harbor project was authorized by the River and Harbor Act of 1930 (US Army Engineer District (USAED), San Francisco, 1979), and construction was completed in 1961. It consists of a jettied entrance at the river mouth; a 10-ft-deep, 100-ft-wide entrance channel; and a 10-ft-deep, 150-ft-wide river channel extending upstream about 0.6 mile. Noyo Mooring Basin is located on the south bank of the river at the upstream limit of the dredged river channel. Also, further upstream, approximately 1.1 miles from the river mouth, a privately owned harbor (Dolphin Isle Marina), is located on the south bank. An aerial photograph of the area is shown in Figure 2.

The Problem

3. Noyo Cove is open to the Pacific Ocean and is exposed to large waves generated by local coastal storms accompanied by strong winds (sea) and distant ocean storms with and without local winds (swell). Waves in excess of 20 ft in height approach the cove from the southwest clockwise through northwest directions. Heavy seas sweep across the cove and through the jettied river entrance, making it impassable for entry or departure during these periods. In addition to these adverse wave conditions, the harbor has experienced strong surging problems due to long-period wave energy resulting in damages to small craft moored there. Shoaling in the river channel also is experienced due to the deposition of material brought down the river during the winter rainy season. This causes navigational difficulties in the shallow river channel, particularly upstream of Noyo Harbor. Vessels are subject to damage by grounding and are forced to wait for favorable tide conditions to provide adequate depths.

4. Improvements at Noyo River and Harbor would result in the reduction of boat and harbor damages, a harbor of refuge for vessels during storm activity, increased commercial fish catch, and increases in recreational boating. The project construction would employ local (currently unemployed) labor or area redevelopment. Also the improvements should enhance the overall commercial fishing operation thereby contributing to the local economic base.

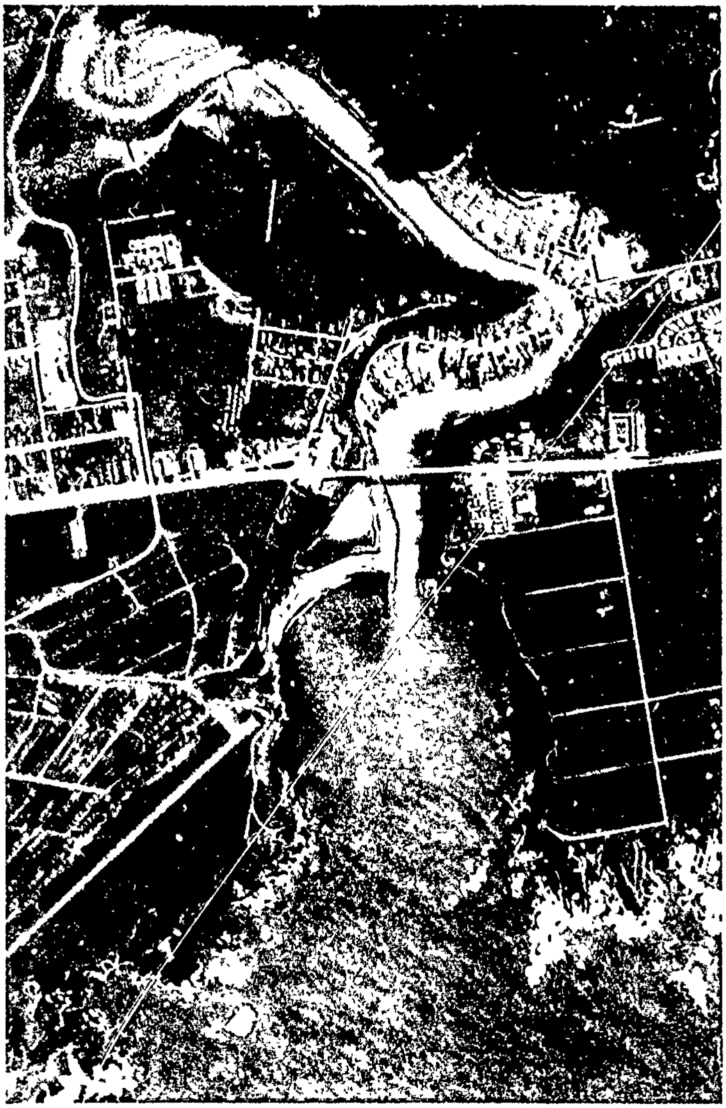

Figure 2. Aerial view of prototype site

Proposed Improvements

5. Authorization for improvements at Noyo River and Harbor was granted by the River and Harbor Act of 1962. Under this authorization, however, breakwaters were proposed to protect the outer cove for development. The breakwaters required were not economically feasible (due to the high cost of construction and maintenance) resulting in the project's being transferred to an inactive category. The Water Resources Development Act (WRDA) of 1976 modified the 1962 project to provide for construction of up to two breakwaters without a specific location to protect the harbor entrance (USAED,

San Francisco, 1979). The location of breakwaters in more shallow water would reduce construction cost significantly. The 1976 WRDA also included additional channel improvements (deepening, widening, and extending) as deemed necessary, subject to applicable economic and environmental criteria.

Previously Reported Model Tests and Conclusions

6. The Noyo River and Harbor model was constructed initially to investigate both short- and long-period wave conditions and river flow conditions in tne river and harbor for comprehensive test conditions. Qualitative information on the effects of the proposed breakwaters on sediment moving down the river also was provided. Details of the investigation were published (Bottin, Acuff, and Markle 1988). Conclusions derived from results of these tests are mentioned below. Plan numbers in the following subparagraphs refer to the previous investigation.

a. Existing conditions are characterized by very rough and turbulent wave conditions in the Noyo River entrance during periods of storm wave attack.

b. Deepening of the entrance channel will not improve wave conditions in the existing river entrance, considering all test conditions.

c. The originally proposed breakwater location (Plan 3) resulted in excessive wave heights (up to 8.8 ft) in the river entrance.

d. Of the 40 expedient rubble-mound breakwater plans (Plans 5 through 42) tested, the alignment of the 637-ft-long breakwater of Plan 39 appeared to be optimum with regard to wave protection, navigation, and economics.

e. The 637-ft-long dolosse breakwater of Plan 43 (same alignment as Plan 39) was selected as the optimum improvement plan for protection of the Noyo River entrance.

f. The breakwater configuration of Plan 43 will result in improved surge conditions due to long-period wave energy in Noyo River and Harbor.

g. The breakwater configuration of Plan 43 will not interfere with the movement of riverine sediment seaward into Noyo Cove, however, the structure will direct sediment to the northern portion of the cove.

Purpose of the Current Investigation

7. At the request of the US Army Engineer District, San Francisco (SPN), the hydraulic model of Noyo River and Harbor was reactivated by the

US Army Engineer Waterways Experiment Station's (WES) Coastal Engineering Research Center (CERC) to determine the optimum breakwater plan that would provide the fishing fleet protection from hazardous wave conditions while traveling through the jettied entrance. The breakwater plan would be developed for 14-ft design waves, as opposed to waves up to 32 ft in the previous study. During storm conditions above a certain threshold (approximately 14-ft waves) fishermen presumably do not go out to fish; therefore, there are less benefits for protecting the entrance under these extreme conditions. Most benefits would be derived for wave conditions with heights of 14 ft or less.

Wave-Height Criterion

8. Completely reliable criteria have not yet been developed for ensuring satisfactory navigation and mooring conditions in small-craft harbors during attack by waves. For this study, however, SPN specified that for an improvement plan to be acceptable, maximum wave heights were not to exceed 6.0 ft (provided the wave was nonbreaking) in the existing Noyo River jettied entrance.

PART II: THE MODEL

Design of Model

9. The Noyo River and Harbor model (Figure 3) was constructed to an undistorted linear scale of 1:75, model to prototype. Scale selection was based on such factors as:

- a. Depth of water required in the model to prevent excessive bottom friction.
- b. Absolute size of model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Available wave-generating and wave-measuring equipment.
- f. Model construction costs.

A geometrically undistorted model was necessary to ensure accurate reproduction of wave and current patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law (Stevens et al. 1942). The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Model-Prototype Scale Relations
Length	L	$L_r = 1:75$
Area	L^2	$A_r = L_r^2 = 1:5,625$
Volume	L^3	$\forall_r = L_r^3 = 1:421,875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$

* Dimensions are in terms of length and time.

10. The proposed breakwaters at Noyo included the use of concrete armor units (dolos). Since the porosity of these armor units differs from that of rock and since the units could not be reproduced to scale (due to cost and time requirements), two-dimensional wave transmission tests were conducted at a scale large enough to have negligible scale effects (i.e. 1:31-scale model) to determine the correct transmission through the proposed structures. This transmission was then duplicated at a scale of 1:75 using rock cross sections

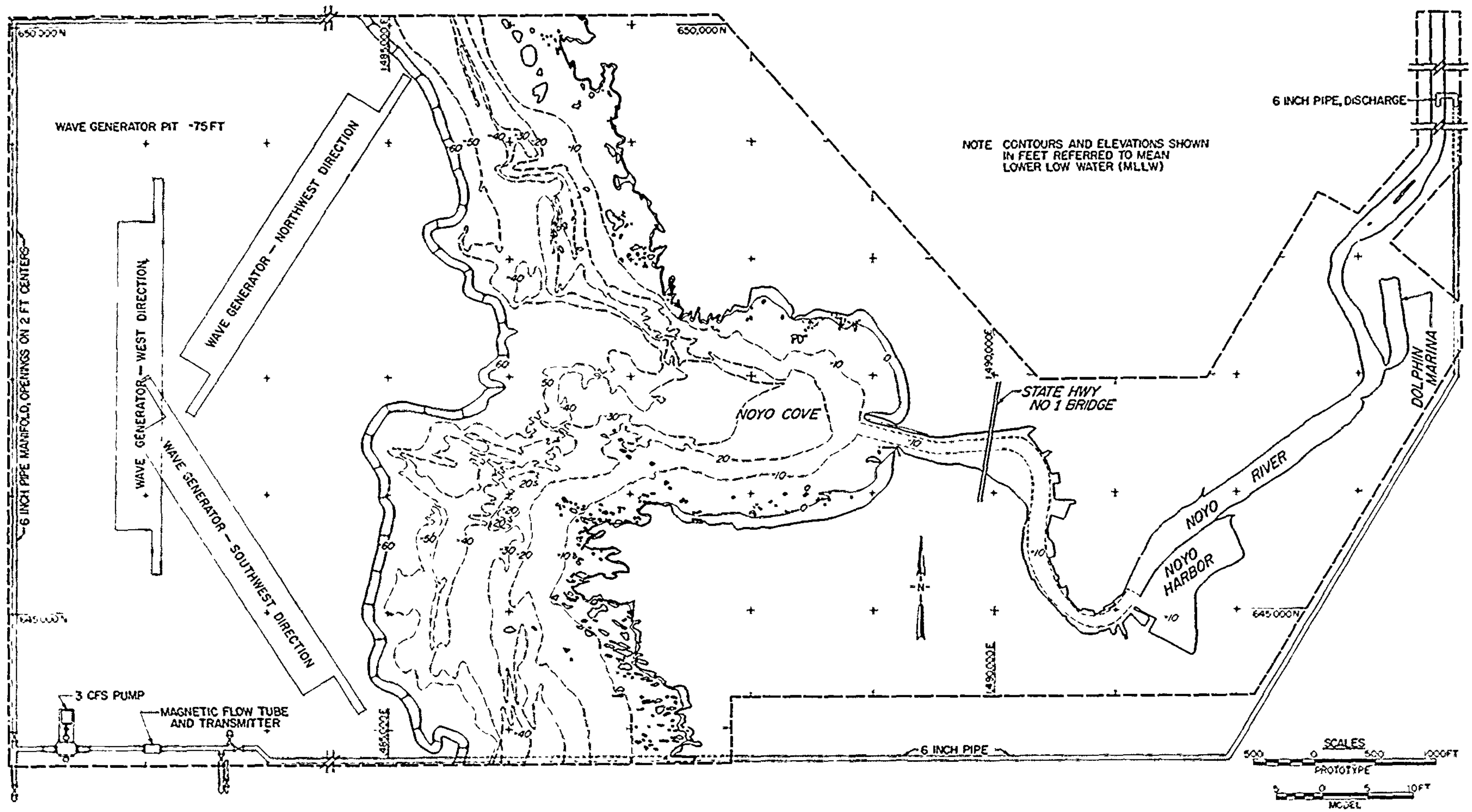

Figure 3. Model layout

and the three-dimensional model structures were built accordingly. These tests are detailed in Bottin, Acuff, and Markle (1988).

11. Parts of the existing jetties at Noyo River entrance are rubble-mound structures. Experience and experimental research have shown that considerable wave energy passes through the interstices of this type of structure; thus, the transmission and absorption of wave energy became a matter of concern in design of the 1:75-scale model. In small-scale hydraulic models, rubble-mound structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures (Le Méhauté 1965). Also, the transmission of wave energy through a rubble-mound structure is relatively less for the small-scale model than for the prototype. Consequently, some adjustment in small-scale model rubble-mound structures is needed to ensure satisfactory reproduction of wave-reflection and wave-transmission characteristics. In past investigations (Dai and Jackson 1966, Brasfeild and Ball 1967) at WES, this adjustment was made by determining the wave-energy transmission characteristics of the proposed structure in a two-dimensional model using a scale large enough to ensure negligible scale effects. A section then was developed for the small-scale, three-dimensional model that would provide essentially the same relative transmission of wave energy. Therefore, from previous findings for structures and wave conditions similar to those at Noyo, it was determined that a close approximation of the correct wave-energy transmission characteristics would be obtained by increasing the size of the rock used in the 1:75-scale model to approximately 1.5 times that required for geometric similarity. Accordingly, in constructing the rubble-mound structures in the Noyo River and Harbor model, the rock sizes were computed linearly by scale, then multiplied by 1.5 to determine the actual sizes to be used in the model.

The Model and Appurtenances

12. The model reproduced the lower 15,000 ft of Noyo River, both Noyo Mooring Basin and Dolphin Isle Marina (located on the south bank), Noyo Cove, approximately 5,500 ft of the California shoreline on each side of the river mouth, and underwater topography in the Pacific Ocean to an offshore depth of 60 ft with a sloping transition to the wave generator pit elevation of -75 ft. The total area reproduced in the model was approximately 12,000 sq ft, representing about 2.4 square miles in the prototype. A general view of the

model is shown in Figure 4. Vertical control for model construction was based on mean lower low water.* Horizontal control was referenced to a local prototype grid system.

Figure 4. General view of model

13. Model waves were generated by a 45-ft-long piston-type generator. The horizontal movement of the piston plate caused a periodic displacement of water incident to this motion. The length of the stroke and the frequency of the piston plate movement were variable over the range necessary to generate waves with the required characteristics. In addition, the wave generator was mounted on retractable casters which enabled it to be positioned to generate waves from the required directions.

14. An Automated Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 5), was used to secure wave-height data at selected locations in the model. Basically, through the use of a minicomputer, ADACS recorded onto magnetic disk the electrical output of parallel-wire, resistance-type wave gages that measured the change in water-surface elevation with respect to time. The magnetic disk output of ADACS then was analyzed to obtain the wave-height data.

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* All elevations (el) cited herein are in feet referred to as mean lower low water (mllw) unless otherwise defined.

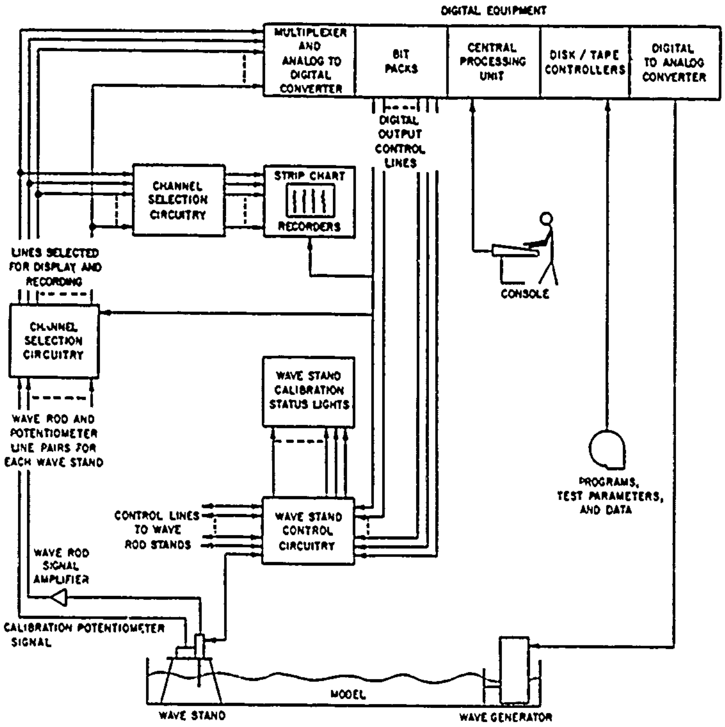

Figure 5. Automated data acquisition and control system

15. A 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter cf t model to dampen any wave energy that might otherwise be reflected from the model walls. In addition, guide vanes were placed along the wave generator sides in the flat pit area to ensure proper formation of the wave train incident to the model contours.

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

16. Still-water levels (swl's) for wave action models are selected so that the various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include the refraction of waves in the project area, the overtopping of structures by the waves, the reflection of wave energy from various structures, and the transmission of wave energy through porous structures.

17. In most cases, it is desirable to select a model swl that closely approximates the higher water stages which normally occur in the prototype for the following reasons:

- a. The maximum amount of wave energy reaching a coastal area normally occurs during the higher water phase of the local tidal cycle.
- b. Most storms moving onshore are characteristically accompanied by a higher water level due to wind tide and shoreward mass transport.
- c. The selection of a high swl helps minimize model scale effects due to viscous bottom friction.
- d. When a high swl is selected, a model investigation tends to yield more conservative results.

Swl's of 0.0 and +7.0 ft were selected by SPN for use during model tests. The lower value (0.0 ft) represents mllw, and the upper value (+7.0 ft) represents a monthly occurrence at the site.

Factors influencing selection of test wave characteristics

18. In planning the testing program for a model investigation of harbor wave-action problems, it is necessary to select dimensions and directions for the test waves that will allow a realistic test of proposed improvement plans and an accurate evaluation of the elements of the various proposals. Surface-wind waves are generated primarily by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interactions between individual wave components. The height and period of the maximum wave that can be generated by a given storm depend on the wind speed, the length of time that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows.

Selection of test wave conditions entails evaluation of such factors as:

- a. The fetch and decay distances (the latter being the distance over which waves travel after leaving the generating area) for various directions from which waves can attack the problem area.
- b. The frequency of occurrence and duration of storm winds from the different directions.
- c. The alignment, size, and relative geographic position of the navigation entrance to the harbor.
- d. The alignments, lengths, and locations of the various reflecting surfaces inside the harbor.
- e. The refraction of waves caused by differentials in depth in the area seaward of the harbor, which may create either a concentration or a diffusion of wave energy at the harbor site.

Wave refraction

19. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to the selection of test wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. The change in wave height and direction are determined by using the numerical Regional Coastal Processes Wave Transformation Model (RCPWAVE) development by Ebersole, Cialone, and Prater (1986). This model predicts the transformation of monochromatic waves over complex bathymetry and includes refractive and diffractive effects. The model is very efficient for modeling large areas of coastline subjected to widely varying wave conditions and, therefore, is an extremely useful tool in the solution of many types of coastal engineering problems.

20. When the refraction coefficient (K_r) is determined, it is multiplied by the shoaling coefficient (K_s) and gives a conversion factor for transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, a function of wave length and water depth, can be obtained from the Shore Protection Manual (1984).

21. Refraction and shoaling coefficients and shallow-water directions were initially obtained at Noyo for various wave periods from five deepwater directions (northwest counterclockwise through southwest). For this study, however, only three directions of wave approach were utilized during model testing. Based on the refracted directions secured at the approximate

locations of the wave generator in the model, the following test directions were selected:

Deepwater Direction deg	Selected Shallow-Water Test Direction deg
West-northwest, 292.5	288
West, 270	270
West-southwest, 247.5	254

These three directions are the predominant directions that cause hazardous navigation conditions within the jetty entrance. Of these directions, waves from the west-northwest occur most frequently.

Prototype wave data and selection of test waves

22. For the initial tests (Bottin, Acuff, and Markle 1988) statistical deepwater wave hindcast data representative of the Noyo area were obtained from the Sea-State Engineering Analysis System (SEAS) by Corson (1985). Deepwater SEAS data were converted to shallow-water values by application of refraction and shoaling coefficients and test wave characteristics with periods ranging from 7 to 19 sec, and heights ranging from 6 to 32 ft were selected for model testing. For the tests reported herein, however, 15-sec, 14-ft test waves were selected by SPN for testing, since limited benefits could be gained for more severe conditions. Some of the most promising improvement plans were also subjected to 15-sec, 20-ft waves to aid in determining the frequency of acceptable wave heights in the jettied entrance for economic benefits.

Analysis of Model Data

23. Relative merits of the various plans tested were evaluated by comparison of wave heights at selected locations in the model and visual observations and wave pattern photographs. In the wave-height data analysis, the average height of the highest one-third of the waves recorded at each gage location was computed. All wave heights then were adjusted to compensate for excessive model wave-height attenuation due to viscous bottom friction by application of Keulegan's equation (Keulegan 1950). From this equation, reduction of wave heights in the model (relative to the prototype) can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel.

PART IV: TESTS AND RESULTS

The Tests

Existing conditions

24. Prior to testing of the improvement plans, tests were obtained for existing conditions (Plate 1). Wave-height data and wave pattern photographs were secured for test waves from the three test directions. Test results provided a base from which to evaluate the merits of the various improvement plans.

Improvement plans

25. Wave-height data and wave pattern photographs were secured for 31 test plan configurations. Various combinations of inner and outer breakwaters, both attached and detached, were tested with variations consisting of changes in lengths, alignments, and locations of the structures. Brief descriptions of the improvement plans are presented in the following subparagraphs; dimensional details are shown in Plates 2 through 10.

- a. Plan 1 (Plate 2) consisted of a 500-ft-long shore-connected outer breakwater attached on the north side of the cove.
- b. Plan 2 (Plate 2) entailed the 500-ft-long shore-connected outer breakwater of Plan 1 with a 400-ft-long detached inner breakwater. The inner breakwater was positioned to form a 300-ft opening between its toe and the toe of the existing north jetty. The north head of the inner breakwater was located on a projection of the existing channel center line.
- c. Plan 3 (Plate 2) involved the elements of Plan 2, but 100 ft of the north end of the inner breakwater was removed, resulting in a 300-ft-long detached structure.
- d. Plan 4 (Plate 2) included the elements of Plan 2 with 200 ft of the north end of the inner breakwater removed, resulting in a 200-ft-long detached structure.
- e. Plan 5 (Plate 3) consisted of the elements of Plan 4 but 100 ft of the outer breakwater was removed which resulted in a 400-ft-long attached breakwater.
- f. Plan 6 (Plate 3) entailed the elements of Plan 4 with 200 ft of the outer breakwater removed, resulting in a 300-ft-long attached breakwater.
- g. Plan 7 (Plate 3) consisted of the 300-ft-long shore-connected outer breakwater of Plan 6 and the detached inner breakwater of Plan 4, but 50 ft of breakwater length was added to the north end of the inner structure, resulting in a 250-ft-long detached breakwater.

h. Plan 8 (Plate 3) entailed the elements of Plan 7 with 50 ft of additional breakwater length added to the north end of the inner structure, resulting in a 300-ft-long detached breakwater.

i. Plan 9 (Plate 4) included the elements of Plan 7 with 100 ft of breakwater length added to the north end of the inner structure, resulting in a 325-ft-long detached breakwater.

j. Plan 10 (Plate 4) involved the elements of Plan 7 with 75 ft of breakwater length added to the north end of the inner structure, resulting in a 325-ft-long detached breakwater.

k. Plan 11 (Plate 5) consisted of the elements of Plan 7 but 100 ft of breakwater length was added to the north end of the inner structure and 50 ft was removed from the south end. This resulted in a 300-ft-long detached breakwater.

l. Plan 12 (Plate 5) entailed the elements of Plan 7 with 100 ft of breakwater length added to the north end of the inner breakwater and 100 ft removed from the south end. The inner breakwater length remained at 250 ft.

m. Plan 13 (Plate 5) included the elements of Plan 7 with 100 ft of breakwater length added to the north end and 150 ft removed from the south end of the inner breakwater, resulting in a 200-ft-long structure.

n. Plan 14 (Plate 5) involved the elements of Plan 7 with 150 ft of breakwater length added to the north end of the inner breakwater and 150 ft removed from the south end. This resulted in an inner breakwater length of 250 ft with the north head positioned on a projection of the existing channel center line. A 300-ft opening existed between the toe of the inner breakwater and the toe of the existing north jetty.

o. Plan 15 (Plate 6) consisted of a 400-ft-long shore-connected north breakwater and a 500-ft-long detached outer south breakwater.

p. Plan 16 (Plate 6) entailed the removal of the Plan 15 shore-connected north breakwater leaving only the 500-ft-long outer detached south breakwater.

q. Plan 17 (Plate 7) consisted of a 600-ft-long outer detached south breakwater. This structure was positioned slightly seaward of the Plan 16 structure.

r. Plan 18 (Plate 7) entailed the outer breakwater of Plan 17 but 200 ft of the structure length was removed from its shoreward end, resulting in a 400-ft-long detached breakwater.

s. Plan 19 (Plate 8) consisted of a 637-ft-long breakwater that originated at the large rock south of the existing south jetty and curved across the entrance. The north head of the curved breakwater terminated on a projection of the north channel limits.

t. Plan 20 (Plate 8) included the elements of Plan 19 with 100 ft of breakwater length removed from the seaward end, resulting in a 537-ft-long structure.

u. Plan 21 (Plate 8) involved the elements of Plan 19, but 137 ft of breakwater length was removed from the seaward end, resulting in a 500-ft-long structure.

v. Plan 22 (Plate 8) entailed the elements of Plan 19 with 187 ft of breakwater length removed from the seaward end of the structure, resulting in a 450-ft-long breakwater. This configuration resulted also in a 250-ft entrance width between the toes of the breakwater and existing north jetty; however, once through the entrance opening, the narrowest width was approximately 200 ft between the toes of the two structures.

w. Plan 23 (Plate 9) consisted of a 400-ft-long detached inner north breakwater and a 250-ft-long detached inner south breakwater. The alignment of the structures resulted in a 250-ft width between the toes of the new breakwaters and a 350-ft width between the toe of the south breakwater and the existing north jetty.

x. Plan 24 (Plate 9) involved the elements of Plan 23, but 75 ft of breakwater length was removed from the north end of the north breakwater which resulted in a 325-ft-long structure.

y. Plan 25 (Plate 9) included the elements of Plan 23, with 75 ft of breakwater length removed from the north end of the north breakwater and added to the south end, of the structure. Its length remained 400 ft.

z. Plan 26 (Plate 9) entailed the elements of Plan 23, but 75 ft of breakwater length was removed from the north end of the north breakwater and 50 ft was added to the south end, resulting in a 375-ft-long structure. The distance between the north and south breakwater toes was approximately 230 ft for this plan.

aa. Plan 27 (Plate 10) consisted of a 250-ft-long detached inner breakwater and a 250-ft-long shore-connected outer north breakwater. The head of the inner breakwater was located on a projection of the existing channel center line, and the width between the toes of the inner breakwater and the existing north jetty was 300 ft.

bb. Plan 28 (Plate 10) included the elements of Plan 27, but 50 ft of the seaward end of the outer breakwater was removed, resulting in a 200-ft-long structure.

cc. Plan 29 (Plate 10) involved the elements of Plan 27 with 100 ft of the seaward end of the outer breakwater removed, resulting in a 150-ft-long structure.

dd. Plan 30 (Plate 10) entailed the elements of Plan 27 with 150 ft of the seaward end of the outer breakwater removed which resulted in a 100-ft-long structure.

ee. Plan 31 (Plate 10) consisted of the elements of Plan 27, but the entire shore-connected outer breakwater was removed. Only the 250-ft-long detached inner breakwater remained.

Wave-height tests and
wave pattern photographs

26. Wave-height tests and wave pattern photographs for the various improvement plans were obtained using test waves from one or more of the directions listed in paragraph 21. Tests involving certain improvement plans were limited to the most critical direction of wave approach. The most promising plans of improvement were tested comprehensively for waves from all three directions. Wave-gage locations for each improvement plan are shown in the referenced plates.

Videotape

27. Videotape footage of model tests was secured for existing conditions and the most promising improvement plans depicting the harbor entrance under attack by waves from the various directions. This footage was furnished to SPN for use in briefings, public meetings, etc.

Test Results

28. In evaluating test results, the relative merits of the improvement plans were based on an analysis of measured wave heights in the entrance. Model wave heights (significant wave height or $H_{1/3}$) were tabulated to show measured values at selected locations.

Existing conditions

29. Results of wave-height tests conducted for existing conditions are presented in Table 1 for test waves from the three directions and two swl's. Maximum wave heights obtained were 11.7 ft in the entrance (Gage 1) for test waves from west-southwest with the +7.0 ft swl. Typical wave patterns secured for existing conditions are shown in Photos 1 through 3.

Improvement plans

30. Wave-height data obtained for Plans 1 through 8 are presented in Table 2 for test waves from west-northwest. Maximum wave heights in the existing entrance (Gage 1) were 8.0, 3.2, 4.2, 4.6, 5.3, 7.7, 6.8, and 5.8 ft for Plans 1 through 8, respectively. The 300-ft-long outer north breakwater and 300-ft-long inner breakwater (Plan 8) achieved the 6.0 ft wave-height criterion at the existing entrance with the minimum amount of structure length for test waves from west-northwest. Typical wave patterns for Plans 1 through 8 are shown in Photos 4 through 11 for test waves from west-northwest.

31. Results of wave-height tests for Plans 8 through 13 are presented

in Table 3 for test waves from west. Maximum wave heights were 6.5, 4.6, 6.6, 5.3, 5.2, and 5.8 ft in the existing entrance for Plans 8 through 13, respectively. For test waves from the west, the Plan 13 configuration met the established wave-height criterion with the least amount of structure length. Representative wave patterns for Plans 8 through 13 are shown in Photos 12 through 17 for test waves from west.

32. Wave heights secured for Plans 12 through 14 for test waves from west-southwest are presented in Table 4 Maximum wave heights were 6.9, 7.3, and 5.6 ft for Plans 12 through 14, respectively, in the existing entrance. Only Plan 14 met the established criterion for waves from this direction. Wave patterns for Plans 12 through 14 for test waves from west-southwest are shown in Photos 18 through 20.

33. Wave-height test results for Plan 14 for test waves from west and west-northwest are presented in Table 5, and typical wave patterns are shown in Photos 21 and 22. Plan 14 met the specified criterion in the entrance for these directions. Maximum wave heights obtained were 5.9 and 3.5 ft for the west and west-northwest directions, respectively.

34. Wave-height data secured for Plans 15 and 16 for test waves from west-northwest and/or west are presented in Table 6. Maximum wave heights in the entrance were 8.6 ft for test waves from west-northwest for Plan 15 and 11.2 ft for west test waves with Plan 16 installed. Neither plan met the established wave-height criterion. Typical wave patterns for Plans 15 and 16 are shown in Photos 23 through 25.

35. Wave heights obtained for Plans 17 and/or 18 are presented in Table 7 for test waves from west-northwest, west, and west-southwest. Maximum wave heights obtained in the entrance were 7.4 and 8.3 ft for Plans 17 and 18, respectively. These results indicated that neither plan would meet the 6.0-ft criterion in the entrance. Typical wave patterns for Plans 17 and 18 are shown in Photos 26 through 30.

36. Results of wave-height tests for Plans 19 through 22 are presented in Table 8 for test waves from west-northwest. Maximum wave heights obtained in the existing entrance were 4.[illegible], 5.5, 5.5, and 6.0 ft, respectively, for Plans 19 through 22. All these plans met the established criterion. Typical wave patterns for Plans 19 through 22 are shown in Photos 31 through 34 for test waves from west-northwest.

37. Wave-height data obtained for Plan 22 for test waves from west and west-southwest are presented in Table 9. Maximum wave heights were 5.0 and

4.9 ft in the entrance for test waves from the west and west-southwest directions, respectively. Wave patterns for Plan 22 for test waves from west and west-southwest are shown in Photos 35 and 36.

38. Wave-height test results for Plans 23 through 26 are presented in Table 10 for test waves from west-northwest. In the existing jettied entrance, maximum wave heights were 7.0, 6.9, 4.6, and 5.8 ft for Plans 23 through 26, respectively. Both Plans 25 and 26 met the specified 6.0-ft criterion, but Plan 26 required less structure length. Typical wave patterns with Plans 23 through 26 installed are shown in Photos 37 through 40 for test waves from west-northwest.

39. Wave-height data obtained for Plan 26 for test waves from west and west-southwest are presented in Table 11. Results indicated that Plan 26 met the established criterion for waves from these directions. Maximum wave heights were 4.5 and 5.5 ft for test waves from west and west-southwest, respectively. Wave patterns for Plan 26 for these two directions are shown in Photos 41 and 42.

40. Plans 14, 17, 22, and 26, at this point in the investigation, were some of the most promising plans. They were subjected to 15-sec, 20-ft test waves as presented in Table 12 to aid in determining frequencies of acceptable wave heights in the jettied entrance for analysis of economic benefits. Maximum wave heights in the entrance were 8.6, 12.6, 9.1, and 8.4 ft for Plans 14, 17, 22, and 26, respectively.

41. Wave-height test results for Plans 27 through 31 are presented in Table 13 for test waves from west-northwest. Maximum wave heights were 3.8, 4.2, 4.4, 4.7, and 5.8 ft in the existing entrance for Plans 27 through 31, respectively. All these test plans met the established criterion in the existing entrance, even the total removal of the shore-connected north breakwater (Plan 31). Typical wave patterns for Plans 27 through 31 are shown in Photos 43 through 47 for test waves from west-northwest.

42. Wave heights secured for Plan 31 for test waves from west and west-southwest are presented in Table 14, and typical wave patterns are shown in Photos 48 and 49. Results revealed that Plan 31 met the wave-height criterion. Maximum wave heights were 5.8 and 4.9 ft for the west and west-southwest directions, respectively.

Discussion of test results

43. Results of wave-height tests for existing conditions indicated rough and turbulent wave conditions in the jettied entrance to Noyo River for

design conditions. Wave heights ranged from 9.3 to 11.7 ft depending on direction of approach. Visual observations also indicated breaking wave conditions in the entrance.

44. Wave-height test results for the initial improvement plans with the outer 500-ft-long shore-connected breakwater and the inner 400-ft-long detached breakwater (Plan 2) revealed wave heights in the existing entrance of only 3.2 ft for waves from west-northwest. Initial modifications indicated that the north breakwater could be reduced to 300 ft in length and the inner breakwater to 250 ft in length (Plan 14) and meet the established 6.0-ft wave-height criterion in the existing jettied entrance. The outer north breakwater was very effective for predominant wave conditions from the west-northwest direction resulting in wave heights in the entrance of only 3.5 ft for Plan 14. Further examination of this configuration indicated that the entire north breakwater could be removed and the 250-ft-long inner structure alone (Plan 31) would meet the 6.0-ft criterion. Wave heights in the entrance, however, would increase to 5.8 ft for test waves from west-northwest.

45. Wave heights obtained for the outer north shore-connected breakwater and the outer south detached breakwater (Plan 15) revealed waves in the entrance up to 8.6 ft for test waves from west-northwest. The outer south breakwaters alone (Plans 16 through 18) were not effective in reducing wave heights in the entrance to an acceptable level for test waves from west-northwest and west. Maximum wave heights ranged from 7.4 to 11.2 ft in the existing entrance for test waves for these test plans. The 600-ft-long outer south breakwater of Plan 17 met the criterion only for test waves from west-southwest.

46. Results of wave-height tests with the curved breakwater seaward of the entrance (Plans 19 through 22) indicated that all the plans met the established wave-height criterion. The original 637-ft breakwater length (Plan 19) resulted in 4.0-ft waves, and the shortest 450-ft-long structure (Plan 22) resulted in 6.0-ft waves in the existing entrance for test waves from west-northwest. Plan 22 resulted also in 5.0- and 4.9-ft wave heights in the entrance for test waves from west and west-southwest, respectively.

47. Wave-height data secured for the two inner detached breakwaters (Plans 23 through 26) indicated that the 375-ft-long north and 250-ft-long south breakwaters of Plan 26 would meet the established criterion in the existing entrance. Maximum wave heights ranged from 4.5 to 5.8 ft in the existing ent nce for design wave conditions depending on the direction of

wave approach. Large wave heights between the two new inner breakwaters were measured, however, that ranged from 11.9 to 14.5 ft. These conditions may result in navigational difficulties at this location.

PART V: CONCLUSIONS

48. Based on the results of the hydraulic model investigation reported herein, it is concluded that:

<u>a</u>. Existing conditions are characterized by very rough and turbulent wave conditions in the Noyo River jettied entrance for 15-sec, 14-ft incident design wave conditions. Waves with maximum heights ranging from 9.3 to 11.7 ft will occur in the entrance, depending on direction of wave approach.

<u>b</u>. Of the test plans involving a shore-connected outer north breakwater and a detached inner breakwater (Plans 1 through 14), Plan 14 (300-ft-long outer and 250-ft-long inner breakwaters) will meet the established 6.0-ft wave-height criterion in the existing entrance for design wave conditions from all directions. Wave heights in the entrance for waves from the predominant west-northwest direction will be 3.5 ft or less.

<u>c</u>. Incremental removal of the Plan 14 outer breakwater (Plans 27 through 31) indicated that the 250-ft-long inner breakwater alone (Plan 31) would meet the established criterion for design wave conditions from all directions. Wave heights up to 5.8 ft will exist in the entrance for waves from the predominant west-northwest direction.

<u>d</u>. Neither the outer shore-connected north breakwater and outer detached south breakwater (Plan 15) nor the outer detached south breakwater plans (Plans 16 through 18) will meet the established wave-height criterion for design wave conditions. Maximum wave heights will range from 7.4 to 11.2 ft in the existing entrance for these plans.

<u>e</u>. Of the improvement plans involving a curved breakwater seaward of the existing entrance (Plans 19 through 22), the 450-ft-long structure of Plan 22 will meet the established wave-height criterion for design wave conditions from all directions. Maximum wave heights of 6.0 ft will exist in the entrance for waves from the predominant west-northwest direction.

<u>f</u>. Of the improvement plans involving two inner detached breakwaters (Plans 23 through 26), the 375-ft-long north structure and 250-ft-long south breakwater (Plan 26) will meet the established wave-height criterion in the entrance for design wave conditions from all directions. Wave heights in the existing entrance will be up to 5.8 ft for waves from the predominant west-northwest direction.

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Table 1

Wave Heights for Existing Conditions for 15-sec, 14-ft Test Waves

Direction	swl ft, mllw	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
West-northwest	0 0	7.8	1.3	18.1	9.9	14.5
	+7.0	9.3	1.5	10.9	8.6	7.8
West	0.0	6.3	0.8	15.1	16.2	10.3
	+7.0	10.0	1.7	12.5	13.0	9.6
West-southwest	0.0	6.8	1.2	11.7	12.1	8.5
	+7.0	11.7	1.4	14.5	17.6	9.5

Table 2

Wave Heights for Plans 1 Through 8 for 15-sec, 14-ft Test Waves from West-Northwest

Plan	swl ft, mllw	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
1	0.0	8.0	0.9	9.7	2.1	7.5
	+7.0	5.0	1.0	6.4	2.5	6.4
2	0.0	2.3	0.4	8.9	2.4	2.1
	+7.0	3.2	0.4	6.4	2.6	2.5
3	+7.0	4.2	0.6	7.3	2.6	3.6
4	+7.0	4.6	0.8	7.0	3.0	4.4
5	+7.0	5.3	1.0	8.0	3.9	5.8
6	0.0	7.7	1.0	13.7	5.8	8.8
	+7.0	5.6	1.2	8.5	6.0	7.1
7	0.0	6.8	0.9	13.8	5.7	5.8
8	0.0	5.8	0.7	13.2	4.5	4.5
	+7.0	4.6	0.9	8.8	5.1	5.1

Table 3

Wave Heights for Plans 8 Through 13 for 15-sec, 14-ft Test Waves from West

		Wave Height, ft				
Plan	swl ft, mllw	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
8	0.0	6.5	1.0	15.7	8.5	3.4
	+7.0	5.5	1.1	12.1	8.9	6.0
9	0.0	4.0	0.8	13.9	8.9	2.8
	+7.0	4.6	0.9	10.9	8.7	5.4
10	0.0	6.6	1.0	14.6	9.7	3.3
	+7.0	6.0	1.2	13.0	9.3	6.3
11	0.0	5.3	0.8	14.3	8.5	2.9
	+7.0	4.6	0.9	11.4	9.4	5.2
12	0.0	5.0	0.5	15.0	8.5	2.8
	+7.0	5.2	1.0	12.2	9.8	5.4
13	0.0	4.8	0.7	14.0	8.0	2.6
	+7.0	5.8	1.2	11.0	8.6	6.3

Table 4

Wave Heights for Plans 12 Through 14 for 15-sec, 14-ft Test Waves from West-Southwest

		Wave Height, ft				
Plan	swl ft, mllw	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5A
13	0.0	3.6	0.7	11.2	8.3	7.5
	+7.0	7.3	1.0	14.8	7.8	11.3
12	+7.0	6.9	1.1	15.1	7.7	11.9
14	0.0	2.6	0.4	12.2	7.9	7.9
	+7.0	5.6	0.9	15.4	7.2	11.4

Table 5

Wave Heights for Plan 14 for 15-sec, 14-ft Test Waves from West-Northwest and West

Direction	swl ft, mllw	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5A
West-northwest	0.0	3.2	0.6	12.5	5.8	8.8
	+7.0	3.5	0.6	8.7	5.6	7.8
West	0.0	4.6	0.6	15.4	6.7	13.7
	+7.0	5.9	1.1	12.2	7.8	9.4

Table 6

Wave Heights for Plans 15 and 16 for 15-sec, 14-ft Test Waves

Direction	Plan	swl ft, mllw	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
West-northwest	15	0.0	8.6	1.1	13.3	5.4	8.0
		+7.0	6.5	1.2	7.5	3.3	8.7
West	15	0.0	6.9	0.8	7.7	3.9	5.1
		+7.0	6.9	1.2	7.6	3.8	9.6
	16	0.0	6.1	0.8	8.8	10.3	5.6
		+7.0	11.2	1.5	7.6	13.2	9.9

Table 7

Wave Heights for Plans 17 and 18 for 15-sec, 14-ft Test Waves

			Wave Height, ft			
Direction	Plan	swl ft, mllw	Gage 1	Gage 2	Gage 3	Gage 5
West-northwest	17	0.0	6.9	1.0	16.7	7.4
		+7.0	5.4	1.3	10.0	7.3
	18	0.0	8.0	0.9	12.0	9.8
		+7.0	6.8	1.6	10.2	9.2
West	17	0.0	7.4	0.8	11.9	7.2
		+7.0	6.2	1.1	7.9	11.3
	18	0.0	7.1	0.8	11.6	7.4
		+7.0	8.3	1.3	8.3	11.6
West-southwest	17	0.0	5.4	0.9	6.9	7.4
		+7.0	3.1	0.6	5.0	9.0

Table 8

Wave Heights for Plans 19 Through 22 for 15-sec, 14-ft Test Waves from West-Northwest

		Wave Height, ft			
Plan	swl ft, mllw	Gage 1	Gage 2	Gage 3	Gage 5
19	0.0	4.0	0.5	12.4	11.9
	+7.0	3.9	0.6	11.5	9.5
20	0.0	5.5	0.7	12.8	11.7
21	0.0	5.5	0.8	12.6	12.8
22	0.0	6.0	0.9	11.8	11.1
	+7.0	6.0	1.0	11.4	11.3

Table 9

Wave Heights for Plan 22 for 15-sec, 14-ft Test Waves from West and West-Southwest

Direction	swl ft, mllw	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 5
West	0.0	5.0	0.6	10.5	10.1
	+7.0	3.8	0.6	9.8	8.3
West-southwest	0.0	3.8	0.5	12.1	9.0
	+7.0	4.9	0.6	14.2	10.7

Table 10

Wave Heights for Plans 23 Through 26 for 15-sec, 14-ft Test Waves from West-Northwest

Plan	swl ft, mllw	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
23	0.0	7.0	1.3	14.5	18.3	10.8
	+7.0	4.5	0.9	9.2	10.6	7.7
24	0.0	6.9	1.4	14.8	18.5	11.0
	+7.0	4.6	0.8	10.5	8.9	8.2
25	0.0	4.6	0.6	14.7	19.4	9.1
26	0.0	5.8	1.0	13.2	18.7	10.6
	+7.0	4.1	0.8	8.6	8.2	6.5

Table 11

Wave Heights for Plan 26 for 15-sec, 14-ft Test Waves from West and West-Southwest

Direction	swl ft, mllw	Wave Height, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5
West	0.0	4.5	0.6	14.5	17.8	8.0
	+7.0	3.0	0.4	8.0	8.4	4.0
West-southwest	0.0	5.5	0.8	11.9	16.5	10.7
	+7.0	5.5	0.9	9.1	11.9	7.2

Table 12

Wave Heights for Plans 14, 17, 22, and 26

for 15-sec, 20-ft Test Waves

			Wave Height, ft				
Direction	Plan	swl ft, mllw	Gage 1	Gage 2	Gage 3	Gage 4*	Gage 5
West-northwest	14	0.0	5.3	0.9	12.9	9.0	9.4
		+7.0	5.5	1.4	13.6	8.9	8.4
	17	0.0	9.2	1.8	18.0	--	13.0
		+7.0	10.4	1.5	14.0	--	15.1
	22	0.0	7.7	0.8	20.6	--	14.8
		+7.0	6.3	0.9	12.4	--	13.2
	26	0.0	8.4	1.5	17.9	21.9	13.0
		+7.0	4.5	1.1	10.7	11.7	7.0
West	14	0.0	2.4	0.5	8.8	9.3	6.3
		+7.0	8.6	1.5	20.1	11.7	16.6
	17	0.0	7.0	1.1	15.4	--	6.9
		+7.0	12.6	2.1	13.3	--	10.9
	22	0.0	3.0	0.5	10.7	--	9.3
		+7.0	9.1	1.3	17.6	--	17.2
	26	0.0	4.9	0.8	13.9	14.0	9.0
		+7.0	6.5	1.1	10.1	12.9	9.0
West-southwest	14	0.0	3.6	0.6	11.1	7.7	9.6
		+7.0	5.4	0.8	14.2	8.1	11.0
	17	0.0	7.5	0.9	9.7	--	9.7
		+7.0	7.2	1.5	8.2	--	13.7
	22	0.0	5.1	0.9	10.9	--	10.2
		+7.0	8.2	0.9	15.1	--	14.0
	26	0.0	4.0	0.8	7.9	14.5	6.6
		+7.0	7.8	1.3	11.3	20.6	11.9

* -- indicates no acceptable wave height data.

Table 13

Wave Heights for Plans 27 Through 31 for 15-sec, 14-ft Test Waves from West-Northwest

		Wave Height, ft				
Plan	swl ft, mllw	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5A
27	0.0	3.8	0.8	13.6	7.2	10.9
	+7.0	3.8	0.6	9.9	6.4	8.4
28	0.0	4.1	0.9	13.5	6.9	10.2
	+7.0	4.2	0.7	10.3	7.9	9.0
29	0.0	4.4	0.9	14.8	7.4	11.1
	+7.0	4.1	0.7	10.5	9.1	9.4
30	0.0	4.7	0.8	14.1	9.8	12.0
	+7.0	4.3	0.7	11.1	11.7	9.7
31	0.0	5.8	0.8	15.0	10.0	14.7
	+7.0	5.0	0.7	12.3	9.5	9.7

Table 14

Wave Heights for Plan 31 for 15-sec, 14-ft Test Waves from West and West-Southwest

		Wave Height, ft				
Direction	swl ft, mllw	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5A
West	0.0	5.8	0.8	14.5	13.7	12.5
	+7.0	3.7	0.6	13.2	14.2	10.0
West-southwest	0.0	4.6	0.7	13.8	11.3	10.2
	+7.0	4.9	0.7	14.2	18.3	11.9

Photo 1. Typical wave patterns for existing conditions; 15-sec, 14-ft waves from west-northwest; swl = +7.0 ft

Photo 2. Typical wave patterns for existing conditions; 15-sec, 14-ft waves from west; swl = +7.0 ft

Photo 3. Typical wave patterns for existing conditions; 15-sec, 14-ft waves from west-southwest; swl = +7.0 ft

Photo 4. Typical wave patterns for Plan 1; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 5. Typical wave patterns for Plan 2; 15-sec, 14-ft waves from west-northwest; swl = +7.0 ft

Photo 6. Typical wave patterns for Plan 3; 15-sec, 14-ft waves from west-northwest; swl = +7.0 ft

Photo 7. Typical wave patterns for Plan 4; 15-sec, 14-ft waves from west-northwest; swl = +7.0 ft

Photo 8. Typical wave patterns for Plan 5; 15-sec, 14-ft waves from west-northwest; swl = +7.0 ft

Photo 9. Typical wave patterns for Plan 6; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 10. Typical wave patterns for Plan 7; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 11. Typical wave patterns for Plan 8; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 12. Typical wave patterns for Plan 8; 15-sec, 14-ft waves from west; swl = 0.0 ft

Photo 13. Typical wave patterns for Plan 9; 15-sec, 14-ft waves from west; swl = +7.0 ft

Photo 14. Typical wave patterns for Plan 10; 15-sec, 14-ft waves from west; swl = +7.0 ft

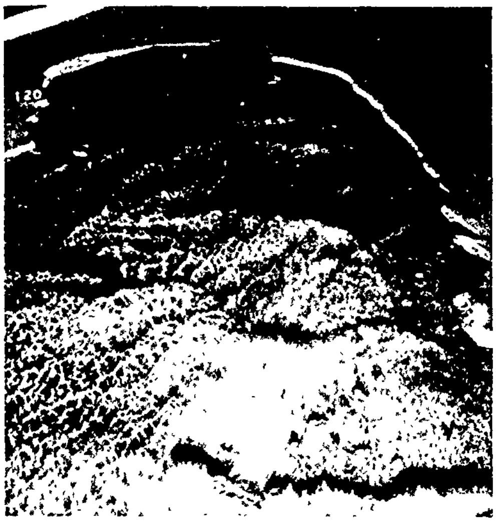


Photo 15. Typical wave patterns for Plan 11; 15-sec, 14-ft waves from west; swl = +7.0 ft

Photo 16. Typical wave patterns for Plan 12; 15-sec, 14-ft waves from west; swl = +7.0 ft

Photo 17. Typical wave patterns for Plan 13; 15-sec, 14-ft waves from west; swl = +7.0 ft

Photo 18. Typical wave patterns for Plan 13; 15-sec, 14-ft waves from west-southwest; swl = +7.0 ft

Photo 19. Typical wave patterns for Plan 12; 15-sec, 14-ft waves from west-southwest; swl = +7.0 ft

Photo 20. Typical wave patterns for Plan 14; 15-sec, 14-ft waves from west-southwest; swl = +7.0 ft

Photo 21. Typical wave patterns for Plan 14; 15-sec, 14-ft waves from west; swl = 0.0 ft

Photo 22. Typical wave patterns for Plan 14; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 23. Typical wave patterns for Plan 15; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 24. Typical wave patterns for Plan 15; 15-sec, 14-ft waves from west; swl = 0.0 ft

Photo 25. Typical wave patterns for Plan 16; 15-sec, 14-ft waves from west; swl = +7.0 ft

Photo 26. Typical wave patterns for Plan 17; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 27. Typical wave patterns for Plan 18; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 28. Typical wave patterns for Plan 17; 15-sec, 14-ft waves from west; swl = +7.0 ft

Photo 29. Typical wave patterns for Plan 18; 15-sec, 14-ft waves from west; swl = +7.0 ft

Photo 30. Typical wave patterns for Plan 17; 15-sec, 14-ft waves from west-southwest; swl = +7.0 ft

Photo 31. Typical wave patterns for Plan 19; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 32. Typical wave patterns for Plan 20; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 33. Typical wave patterns for Plan 21; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 34. Typical wave patterns for Plan 22; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 35. Typical wave patterns for Plan 22; 15-sec, 14-ft waves from west; swl = 0.0 ft

Photo 36. Typical wave patterns for Plan 22; 15-sec, 14-ft waves from west-southwest; swl = +7.0 ft

Photo 37. Typical wave patterns for Plan 23; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 38. Typical wave patterns for Plan 24; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 39. Typical wave patterns for Plan 25; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 40. Typical wave patterns for Plan 26; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 41. Typical wave patterns for Plan 26; 15-sec, 14-ft waves from west; swl = 0.0 ft

Photo 42. Typical wave patterns for Plan 26; 15-sec, 14-ft waves from west-southwest; swl = +7.0 ft

Photo 43. Typical wave patterns for Plan 27; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 44. Typical wave patterns for Plan 28; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

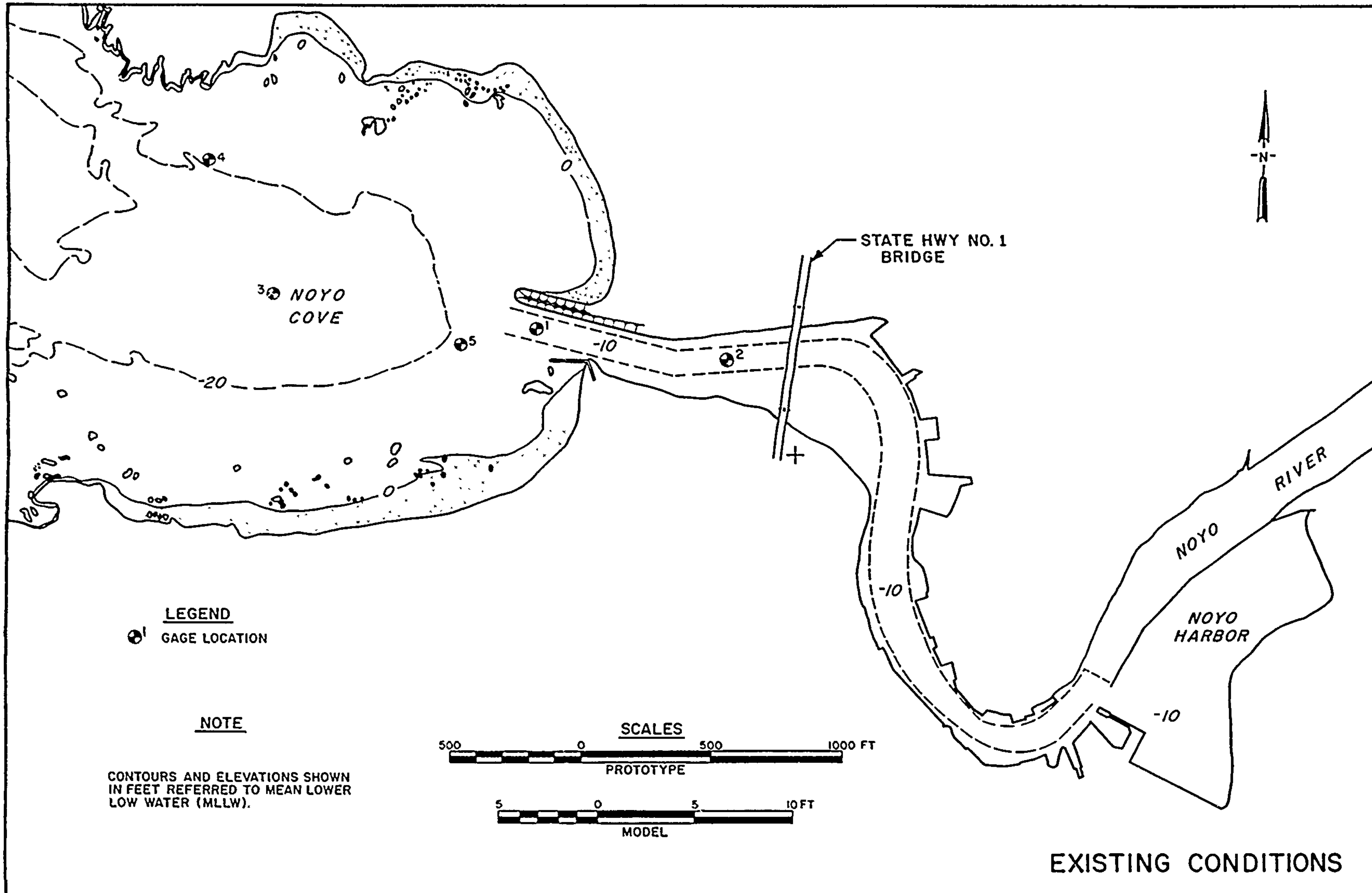
Photo 45. Typical wave patterns for Plan 29; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 46. Typical wave patterns for Plan 30; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

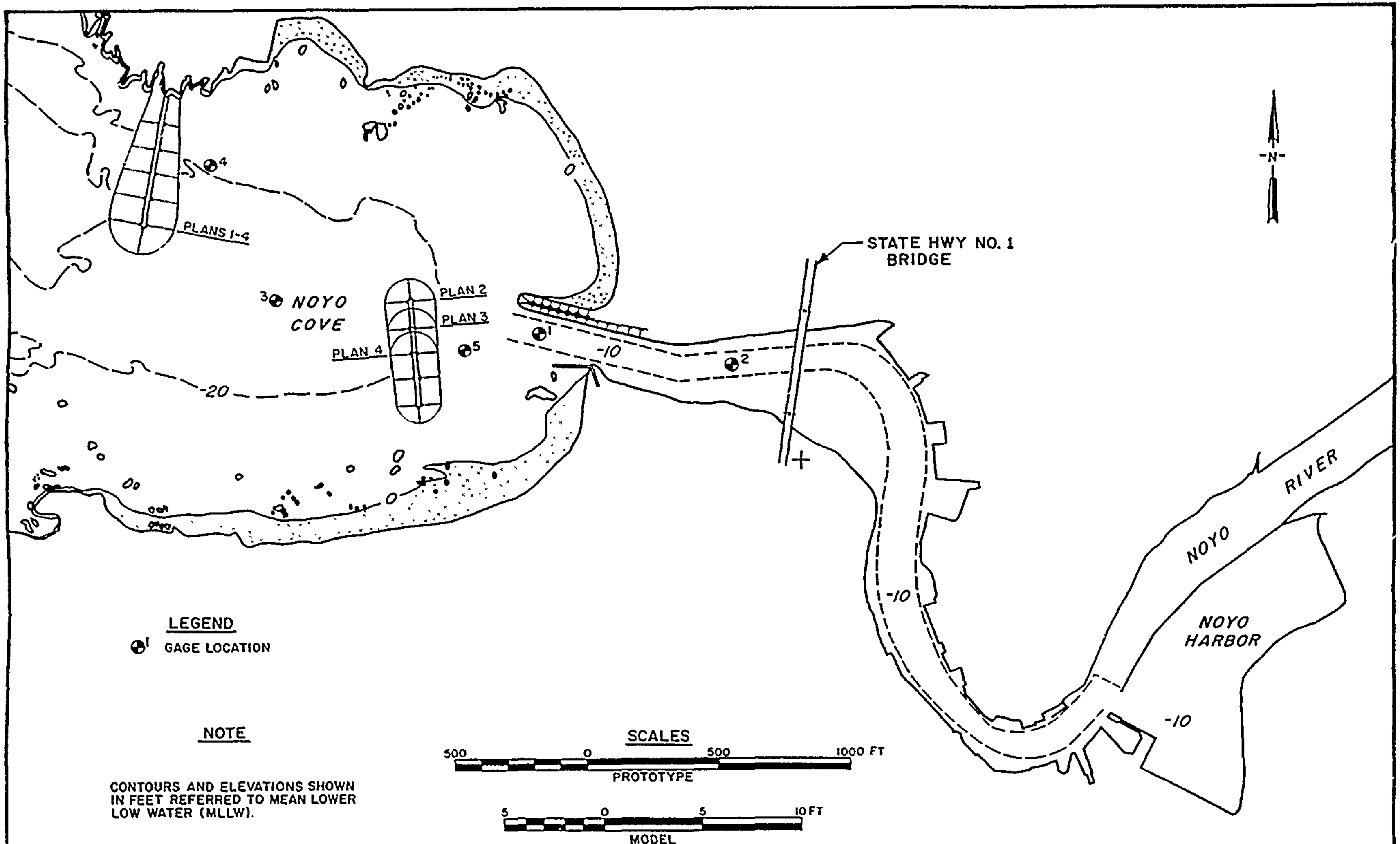
Photo 47. Typical wave patterns for Plan 31; 15-sec, 14-ft waves from west-northwest; swl = 0.0 ft

Photo 48. Typical wave patterns for Plan 31; 15-sec, 14-ft waves from west; swl = 0.0 ft

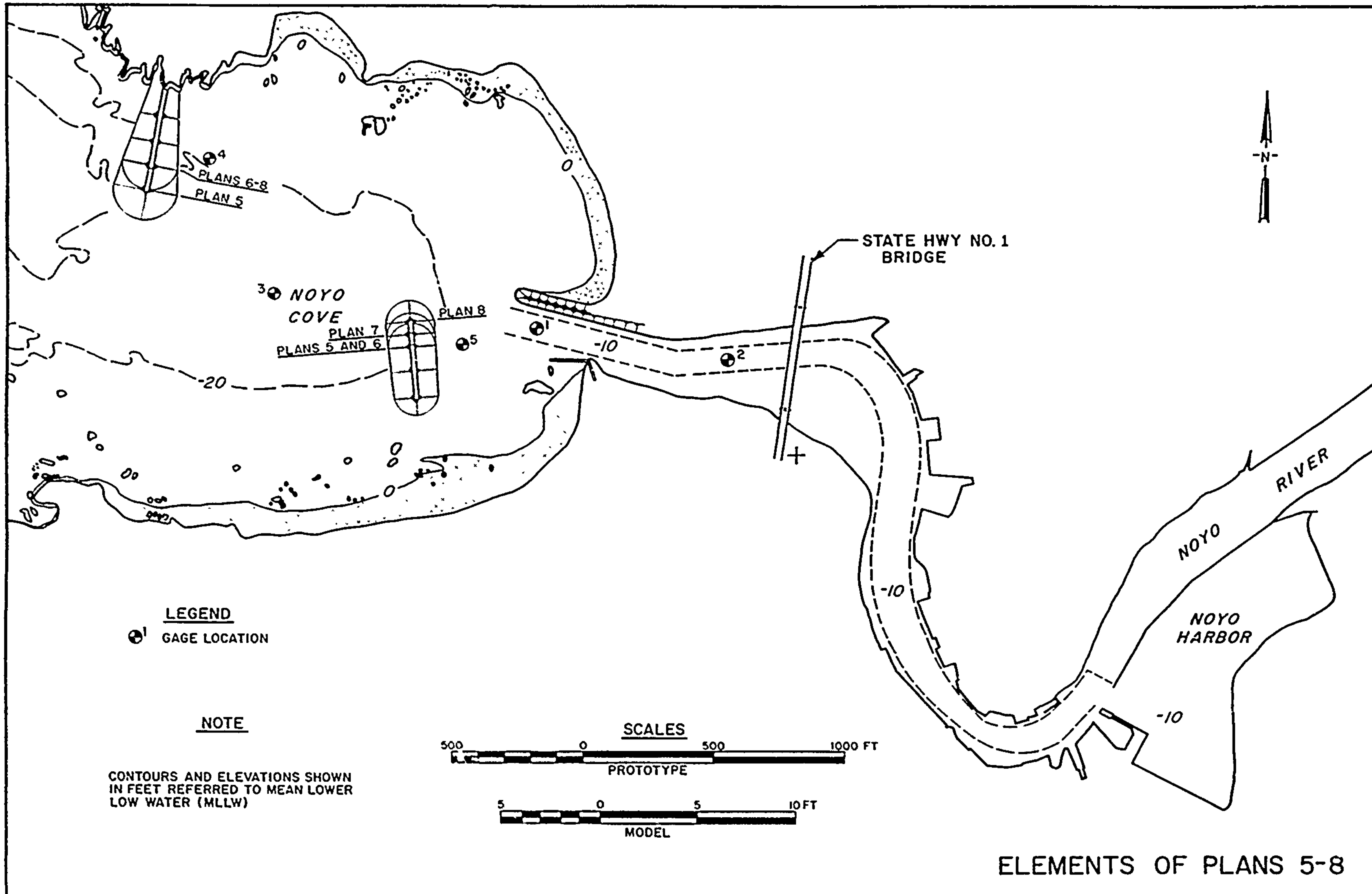
Photo 49. Typical wave patterns for Plan 31; 15-sec, 14-ft waves from west-southwest; swl = 0.0 ft



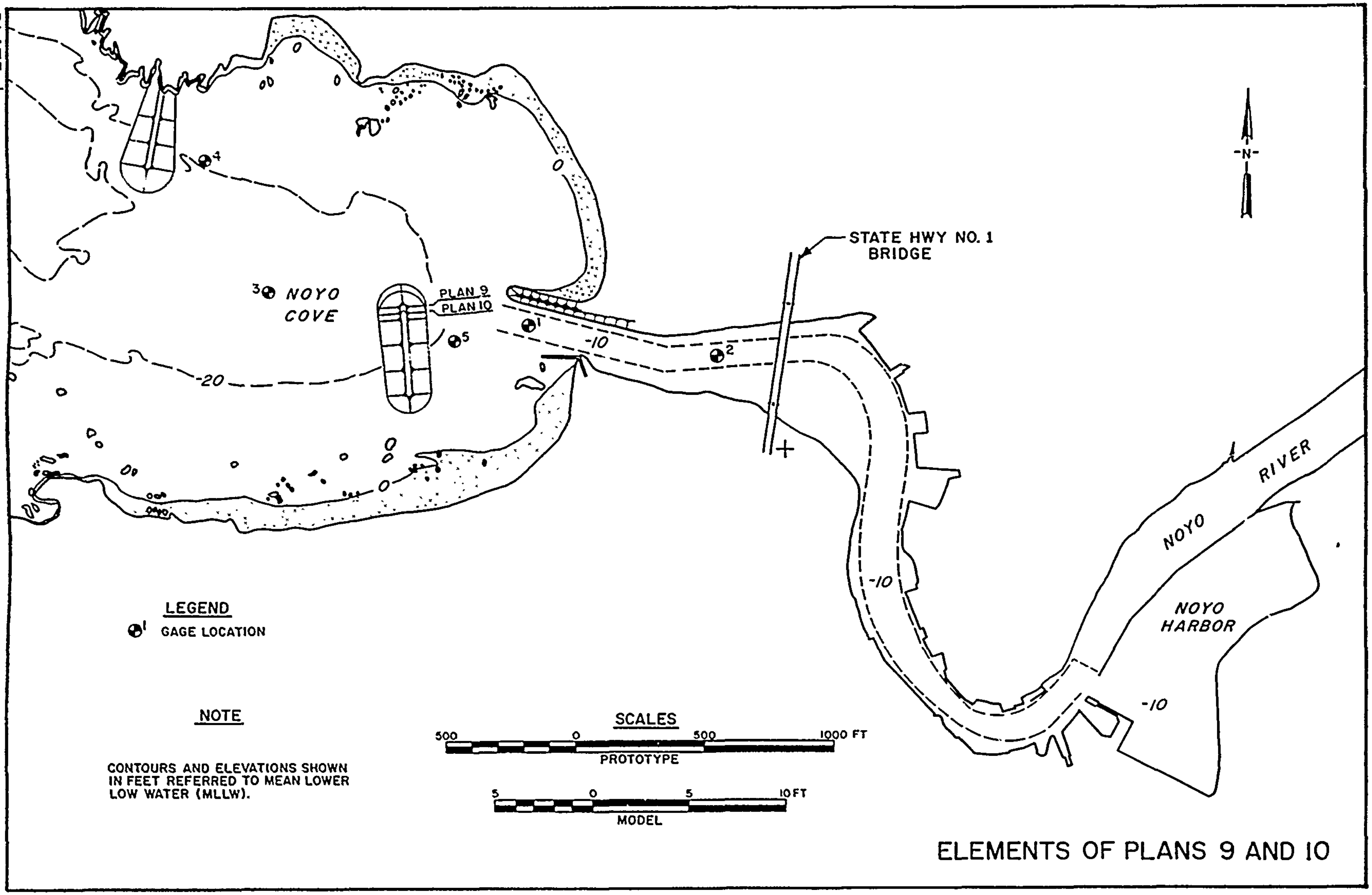
EXISTING CONDITIONS



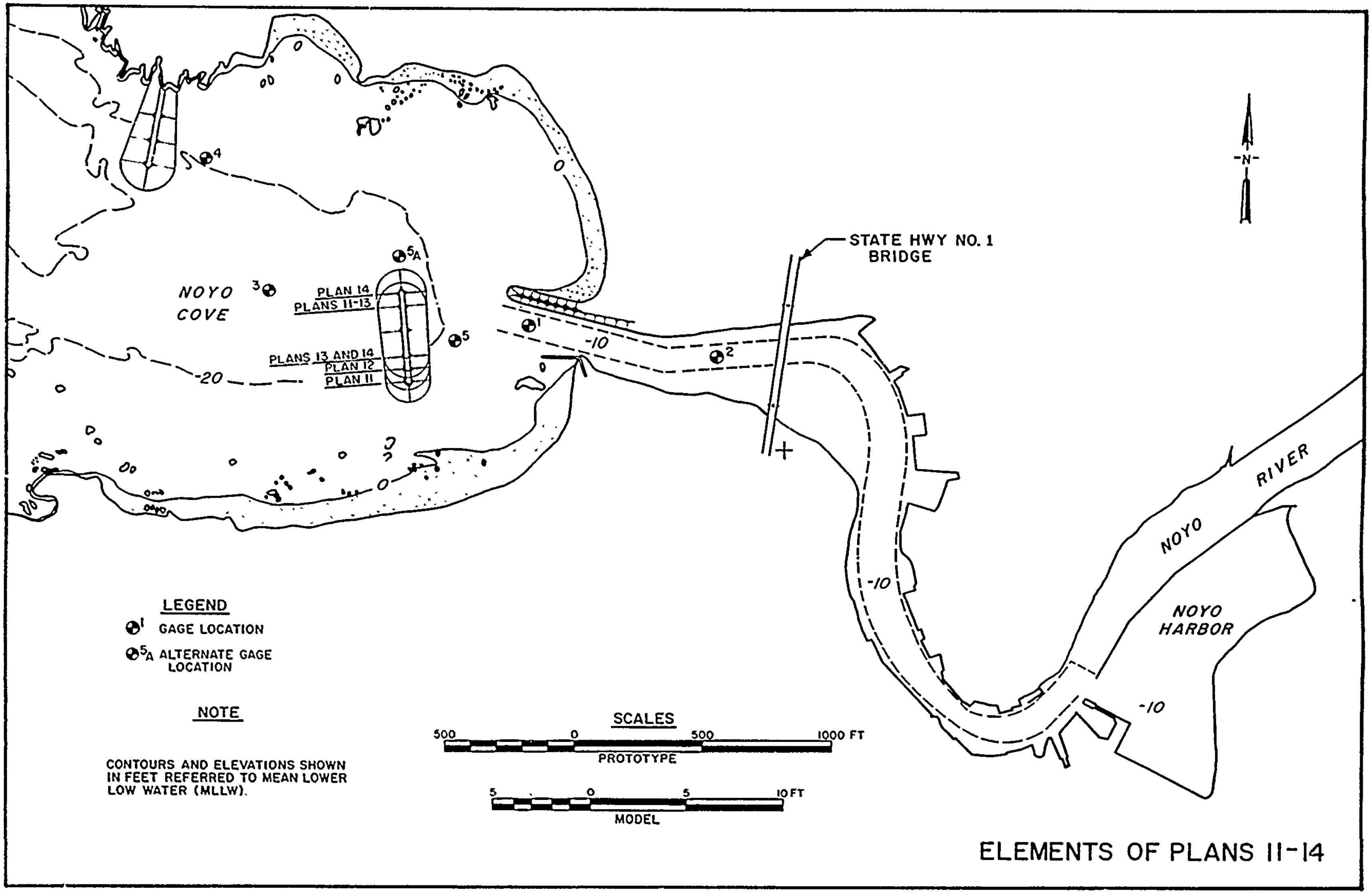
ELEMENTS OF PLANS 1-4



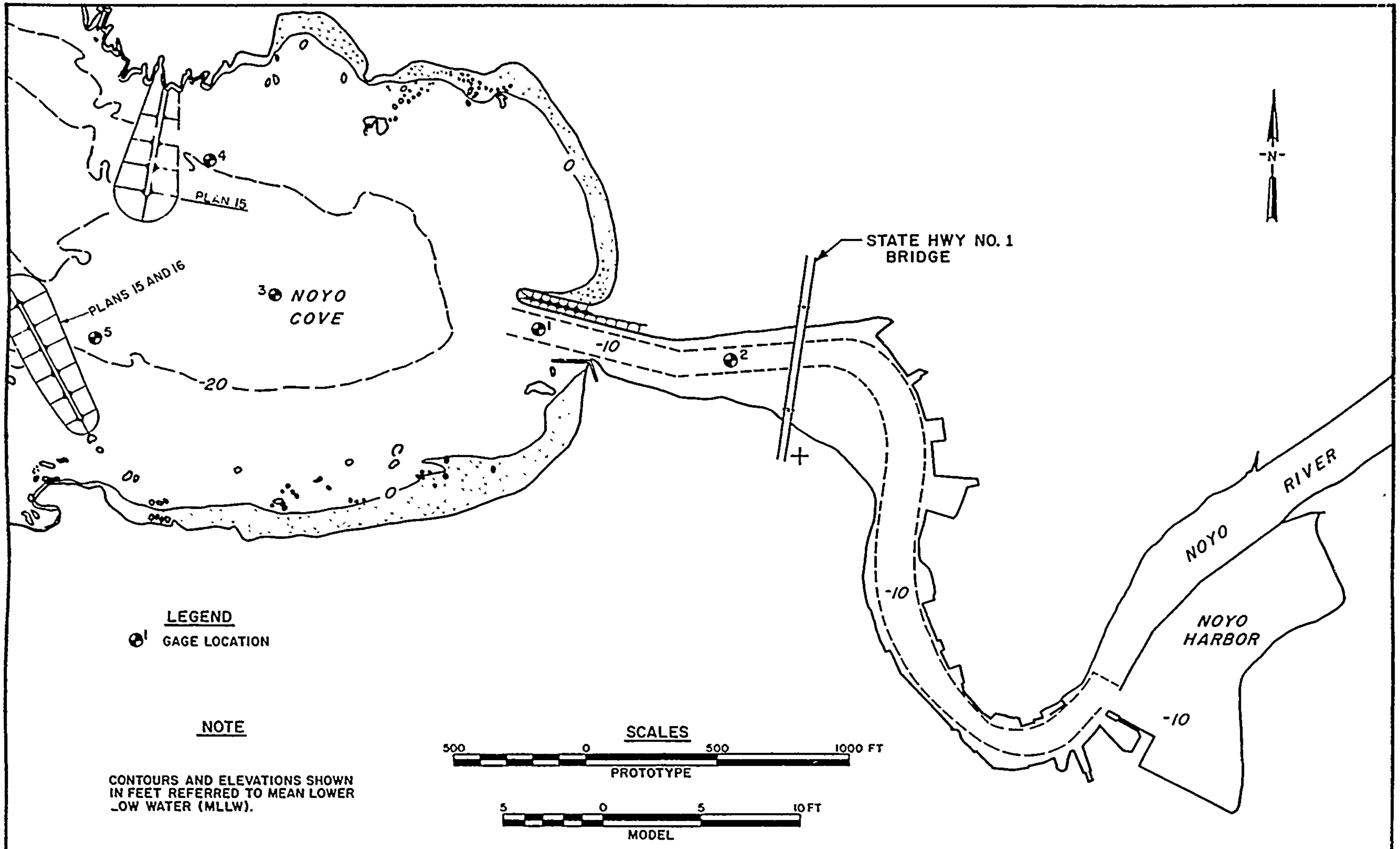
ELEMENTS OF PLANS 5-8



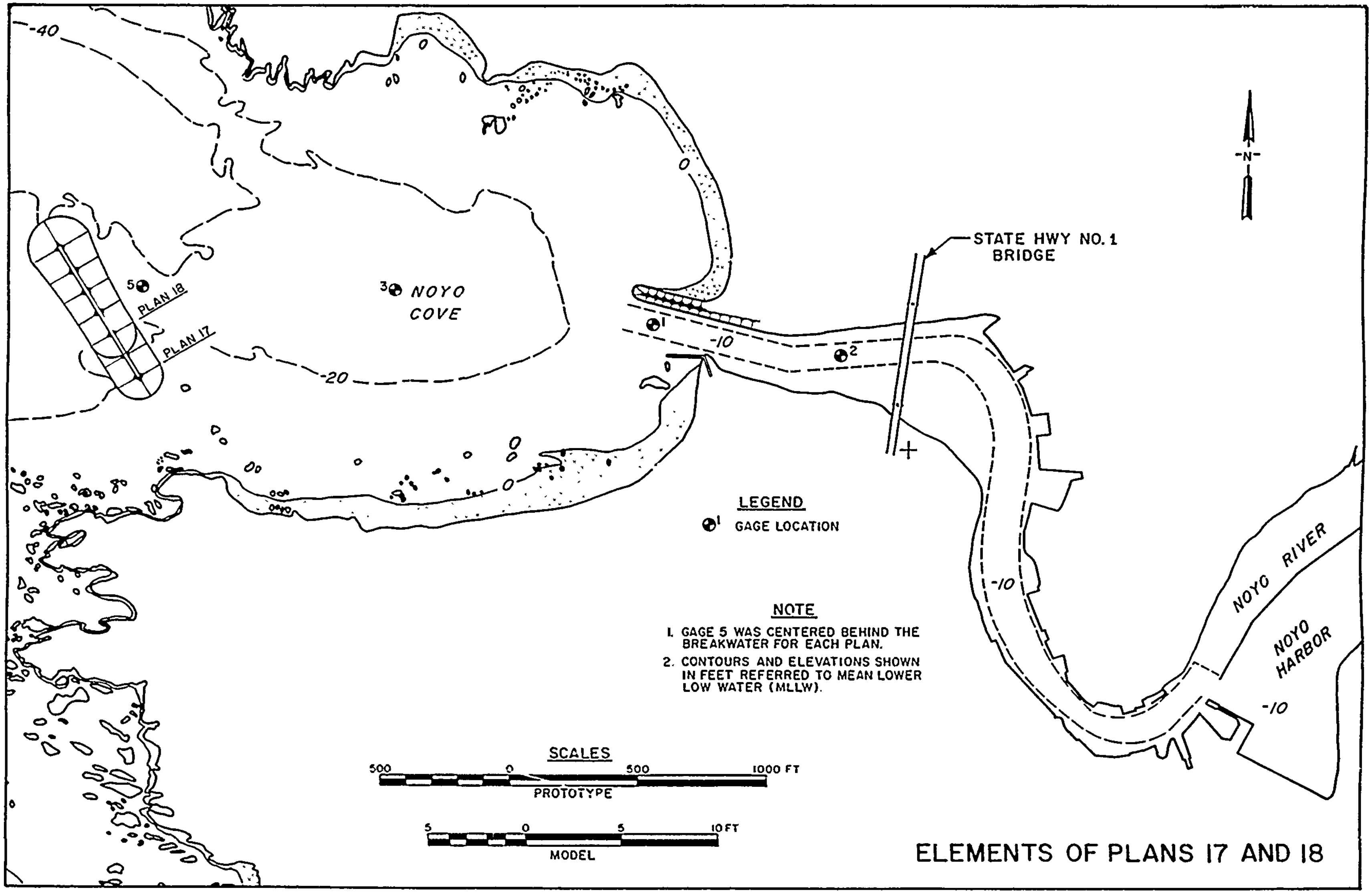
ELEMENTS OF PLANS 9 AND 10



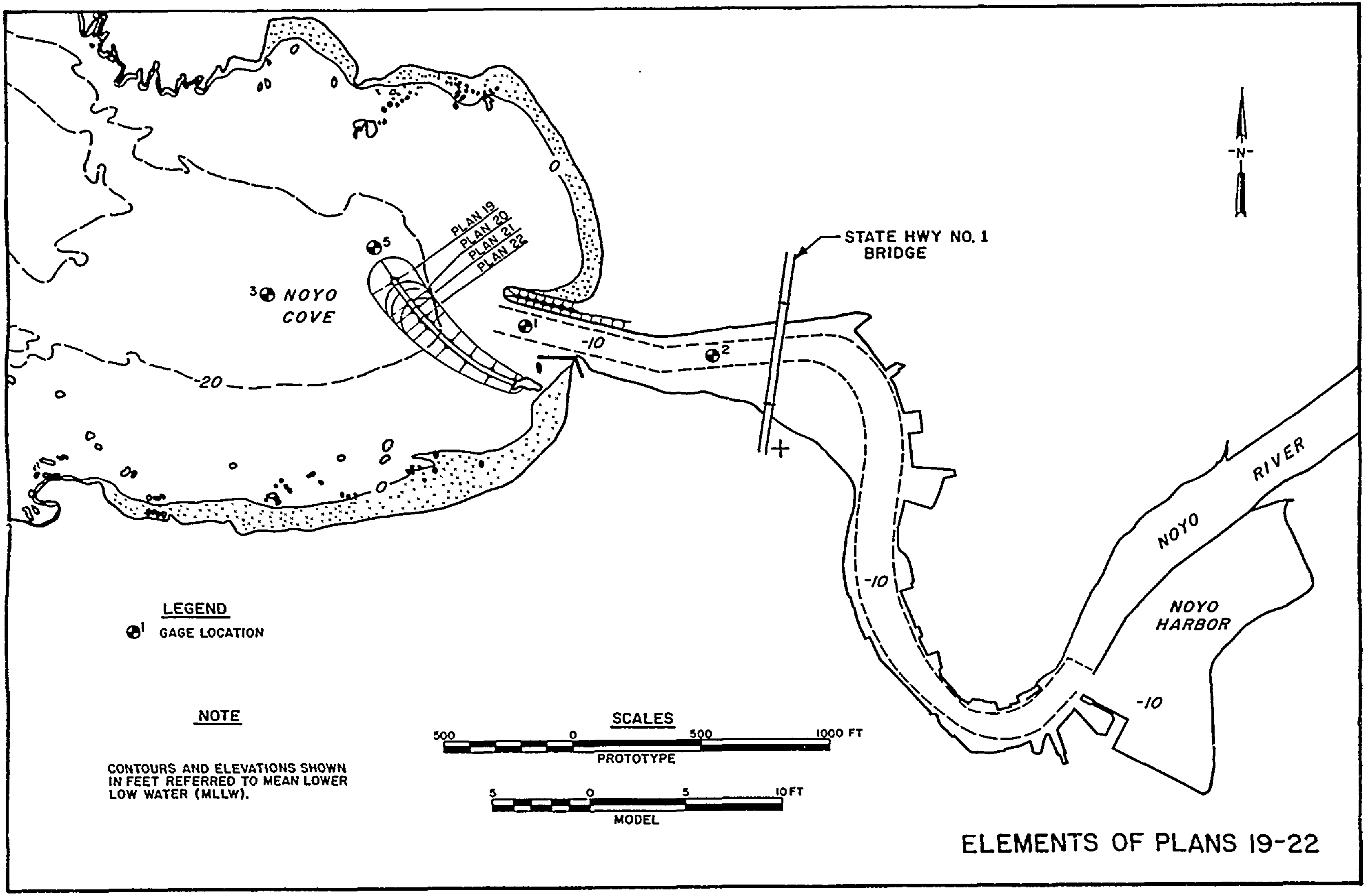
ELEMENTS OF PLANS 11-14



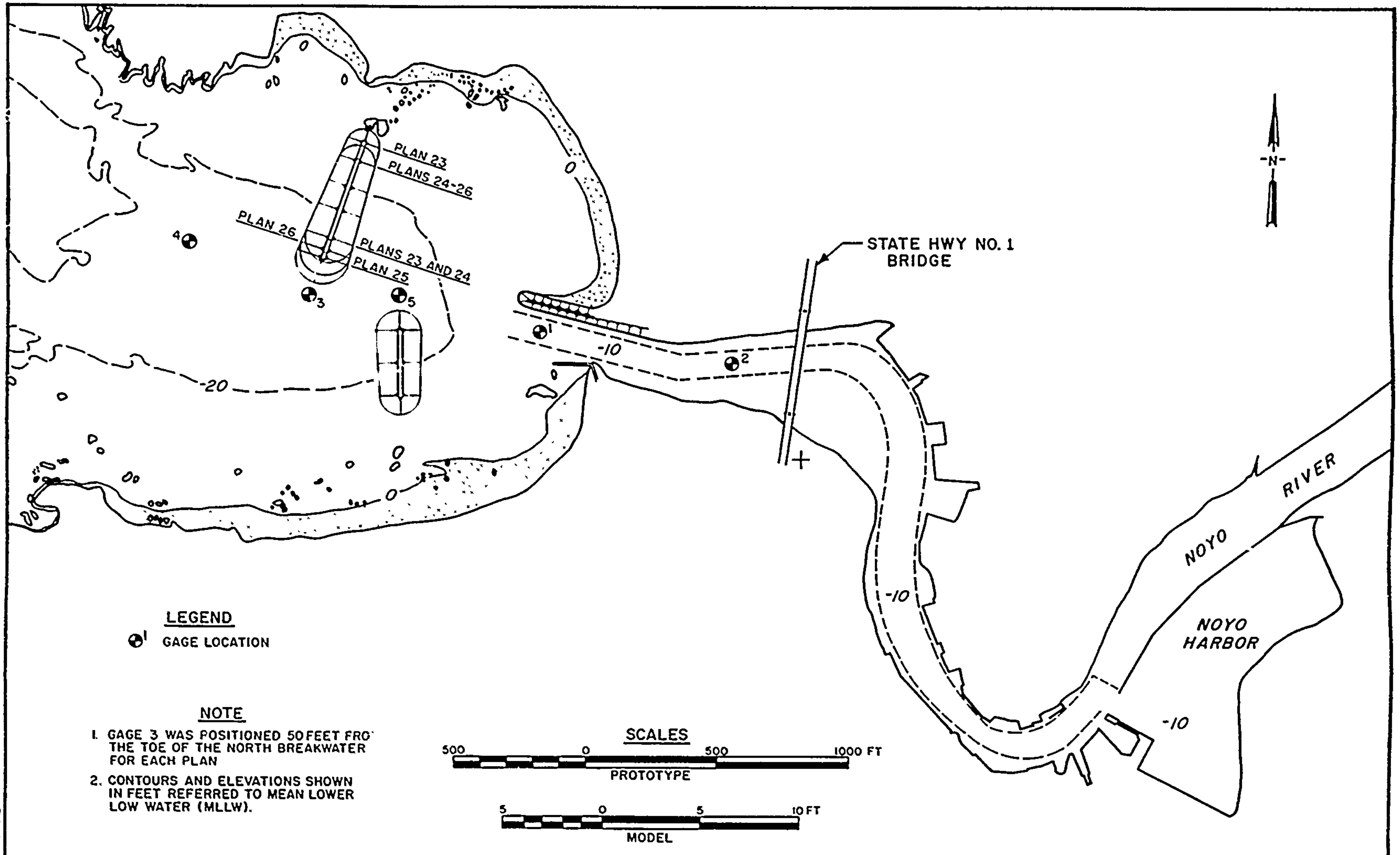
ELEMENTS OF PLANS 15 AND 16



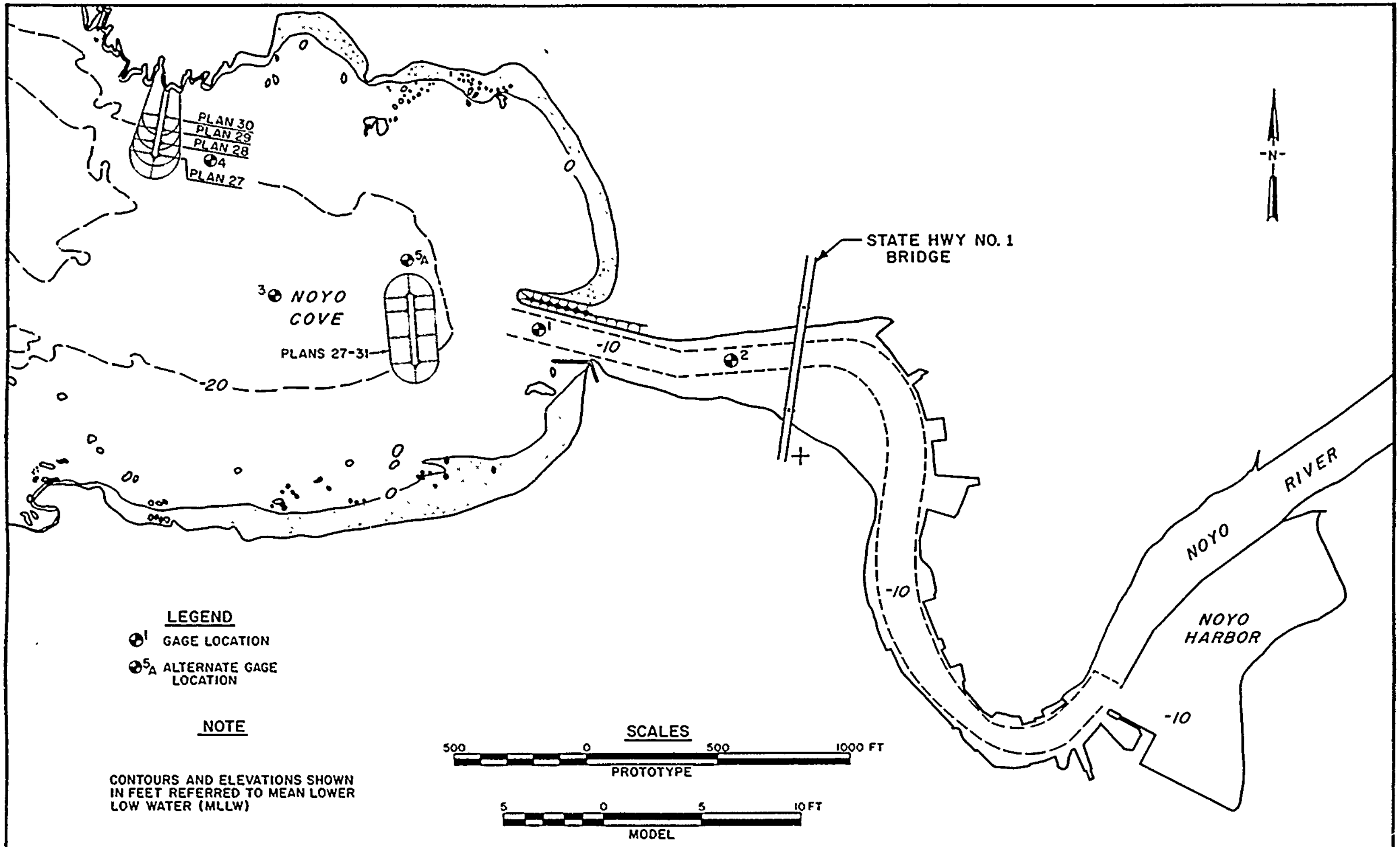
ELEMENTS OF PLANS 17 AND 18



ELEMENTS OF PLANS 19-22



ELEMENTS OF PLANS 23-26



ELEMENTS OF PLANS 27-31